

THE NWS SIMPLIFIED DAM-BREAK FLOOD FORECASTING MODEL

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Keywords: Dam-break Floods, Dam Failure, Emergency Preparedness, Flood Forecasting, Hydraulics, Modelling, Simplified Routing.

ABSTRACT

A simplified model for predicting the downstream flooding produced by a dam failure is presented. This model has been developed to aid flash flood hydrologists who, because warning response time is short or adequate computing facilities are not available, are unable to utilize the NWS Dam-Break Flood Forecasting Model. Using only a hand-held calculator and the dimensionless graphs presented in this paper, the hydrologist may within minutes produce forecasts of the dam-break floodwave peak stages, discharges, and travel times.

The model consists of three major steps. In the first step, the user approximates the river valley as a prismatic channel, the shape of which may be expressed by the relation: $TOPWIDTH = Kh^m$, where h is depth and K and m are the fitted coefficients. The second step entails the calculation of the maximum outflow discharge and stage produced by a time-dependent, rectangular-shaped breach of the dam. In the third step, routing parameters are calculated and dimensionless graphs consulted to route the maximum outflow to selected downstream locations. Data requirements for the simplified model include: 1) cross-sectional information, slope, and Manning's n for the river valley, 2) reservoir storage volume at time of failure, 3) final breach width, depth, and formation time, and 4) the distance from the dam to the downstream points of interest.

The ability of this model to predict the extent and timing of downstream inundation is tested with good results using data obtained during the dam-break floods produced by the failures of Teton Dam and the Buffalo Creek coal-waste dam. Prediction accuracy and the time required to use the model are examined for each application. A step-by-step example application is also presented.

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I INTRODUCTION

The devastation that occurs as impounded reservoir water escapes through the breach of a failed dam and rushes downstream is quick and lethal. This potential for disastrous flash flooding poses a grave threat to many communities located downstream of dams. Indeed, a report by the U.S. Army (1975) indicates 20,000 dams in the U.S. are "so located that failure of the dam could result in loss of human life and appreciable property damage ..." This report, as well as the tragic destruction resulting from the failures of the Buffalo Creek coal-waste dam, the Toccoa Dam, the Teton Dam, and the Laurel Run Dam, underscores the real need for accurate and prompt forecasting of dam-break flooding.

Advising the public of downstream flooding during a dam failure emergency is the responsibility of the National Weather Service (NWS). To aid NWS flash flood hydrologists in forecasting the inundation resulting from dam-failures, the numerical DAMBRK Model (Fread 1977, 1980) was developed for use with high-speed computers to model the outflow hydrograph produced by a time-dependent, partial dam breach and route this hydrograph downstream using the complete one-dimensional unsteady flow equations while accounting for the effects of downstream dams, bridges, and off-channel storage. However, in some situations the use of the DAMBRK Model may be precluded because warning response-time is short or adequate computing facilities are not available.

To alleviate this potential problem and to improve upon the accuracy and versatility of existing simplified dam breach modelling procedures (Saakas and Strelkoff, 1973; McQuivey and Keefer, 1975; Snyder, 1977; and SCS, 1979), the NWS has developed the Simplified Dam-Break (SMPDBK) Flood Forecasting Model. Employing only a hand-held calculator and dimensionless graphs, the user may within minutes produce forecasts of the dam-break floodwave peak discharges, stages, and travel times. It should be noted here however, that the use of the NWS SMPDBK Model is not limited to NWS flash flood hydrologists. Planners, designers, civil defense officials, and consulting engineers who are concerned with the potential effects of a dam failure and who have limited time, resources, data, computer facilities, and/or experience with unsteady flow models may also wish to employ the model to delineate the areas facing danger in a dam-break emergency.

This paper presents an outline of the NWS SMPDBK Model's theoretical basis, some examples of its predictive capabilities, and the procedure for using the model to forecast dam-break floods.

II MODEL DEVELOPMENT

The SMPDBK Model retains the critical deterministic components of the numerical DAMBRK model while eliminating the need for advanced computer facilities. SMPDBK accomplishes this by approximating the downstream channel as a prism, concerning itself with only the peak flows and stages, and utilizing dimensionless graphs developed using

the NWS DAMBRK model. The applicability of the SMPDBK Model is further enhanced by its minimal data requirements; the peak flow at the dam may be calculated with only four readily accessible data values and the downstream channel may be specified by a single "average" cross-section.

Three steps make up the procedure used in the SMPDBK Model. These are: (1) approximation of the channel downstream of the dam as a prismatic channel; (2) calculation of the peak outflow at the dam using the temporal and geometrical description of the breach and the reservoir volume; and (3) calculation of dimensionless routing parameters used with dimensionless routing curves to determine the peak flow and time-to-peak at specified forecast points downstream of the dam.

2.1 Channel Description

The river channel downstream of the dam is approximated as a prismatic channel by defining a single (distance weighted) cross-section and fitting a topwidth as a power law function of depth similar to, but further expanded than the function used by Saakas and Strelkoff (1973) to describe the cross-section. Informal discussions with potential users of this model indicate that this method will adequately describe most channel geometries while satisfying the prismatic requirement. A prismatic channel allows easy calculations of flow area and volume in the downstream channel which is required to accurately predict the amount of hydrograph modification (attenuation and lag).

Approximating the channel as a prism requires three steps. First, topwidth vs. depth data must be obtained from topographic maps or survey notes. For each depth (h_i), a distance weighted topwidth is defined using the relation:

$$\bar{B}_i = \frac{\left[\frac{(B_{i,1} + B_{i,2})}{2} (X_2 - X_1) + \dots + \frac{(B_{i,J-1} + B_{i,J})}{2} (X_J - X_{J-1}) \right]}{(X_J - X_1)} \quad (1)$$

where: h_i is the i^{th} depth, $i = 1, 2, 3 \dots I$
 $B_{i,j}$ is the i^{th} topwidth (corresponding to the i^{th} depth h_i)
at the j^{th} cross-section where $j = 1, 2, 3, \dots J$
 \bar{B}_i is the weighted i^{th} topwidth
 X_j is the downstream distance to the j^{th} cross-section

Defining a topwidth \bar{B}_i for each h_i produces a table of values that may be used for fitting (using least-squares or a log-log plot) a single equation of the form $B = Kh^m$ to define the prismatic channel geometry. The fitted parameters K and m are computed as follows:

$$m = \frac{\frac{\sum \left[(\log h_i) (\log \bar{B}_i) \right]}{I} - \left(\frac{\sum \log h_i}{I} \right) \left(\frac{\sum \log \bar{B}_i}{I} \right)}{\frac{\left(\sum \log h_i \right)^2}{I} - \sum \left(\log h_i \right)^2} \quad (2)$$

$$\log K = \frac{\sum \log \bar{B}_i}{I} - m \left(\frac{\sum \log h_i}{I} \right) \quad (3)$$

$$K = 10^{(\log K)} \quad (4)$$

For rivers with very steep valley side-walls or very dense vegetation adjacent to the channel (see Fig. 1a), an additional parameter (h_v) may be specified to indicate the depth at which the channel geometry no longer follows the $B = Kh^m$ relation. As can be seen in Fig. 1b, this feature allows for a more accurate representation of the true channel-valley shape.

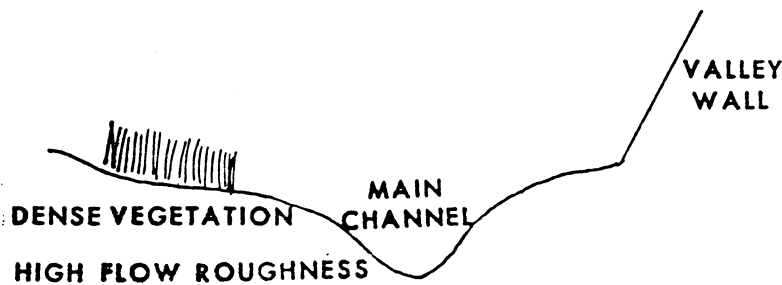


Fig.1a Typical Downstream Cross-Section

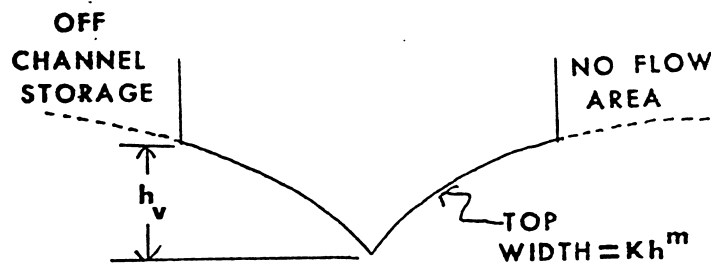


Fig.1b Approximated Prismatic Downstream Cross-Section

As indicated, this equation gives the maximum head over the weir at $t = t_f$. Thus, the maximum breach outflow (Q_{bmax}) occurring at $t = t_f$ may be calculated by substituting this expression for $h_{weir(max)}$ into Eq. (5), i.e.,

$$Q_{bmax} = 3.1 B_r \left(\frac{C}{t_f + \frac{C}{\sqrt{H}}} \right)^3 \quad (13)$$

2.4 Maximum Stage Calculation

Once the maximum outflow has been calculated, the maximum stage (h_{max}) at the downstream face of the dam must be determined. To do this, the flow (Q_v) that produces the stage (h_v) shown in Fig. 3 must first be evaluated and compared with Q_{bmax} to determine whether the maximum stage is above or below h_v . The flow Q_v is evaluated using the Manning equation, i.e.,

$$Q_v = \frac{1.49}{n} S^{1/2} A_v \left(\frac{A_v}{B_v} \right)^{2/3} \quad (14)$$

For the prismatic channel, B_v is given by $K h_v^m$ and A_v is given by $(K h_v^{m+1})/(m+1)$. Substituting these expressions for B_v and A_v into Eq. (14) and simplifying gives the following:

$$Q_v = \frac{1.49}{n} S^{1/2} \left(\frac{K}{(m+1)^{5/3}} \right) h_v^{(m+5/3)} \quad (15)$$

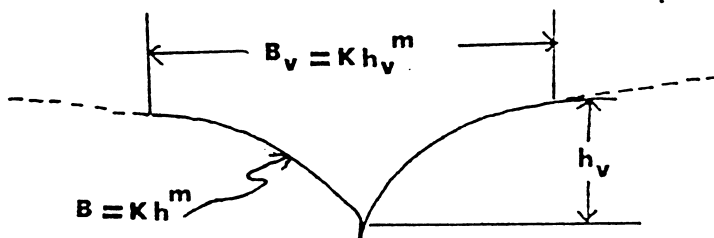


Fig.3 Calculation of Flow (Q_v) For Determining Maximum Stage

If Q_v is found to be greater than Q_{bmax} , then h_v must be greater than h_{max} , and h_{max} is determined using the relation:

$$h_{max} = \left(\frac{Q_{bmax}}{a} \right)^{1/b} \quad (16)$$

where:

$$a = \frac{1.49}{n} S^{1/2} \frac{K}{(m+1)^{5/3}} \quad (17)$$

2.2 Breach Description

As indicated by Fread (1980), most investigators of dam-break flood waves have assumed that the breach or opening formed in a failing dam encompassed the entire dam and occurred instantaneously. While this assumption may be valid for a few concrete arch dams, it is not valid for the exceedingly large number of earth dams. Because earthen dams generally do not fail completely nor instantaneously, the SMPDBK Model allows for the investigation of partial failures occurring over a finite interval of time. And, although the model assumes a rectangular-shaped breach, a trapezoidal breach may be analyzed by specifying a rectangular breach width that is equal to the average width of the trapezoidal breach. Failures due to overtopping of the dam and/or failures in which the breach bottom does not erode to the bottom of the reservoir, may also be analyzed by specifying an appropriate "H" parameter (see Fig. 2) which is the elevation of the reservoir water surface elevation when breach formation commences minus the final breach bottom elevation.

2.3 Maximum Breach Outflow Calculation

The flow (Q_b) in ft^3/sec through the breach of a failed dam may be expressed as broad-crested weir flow, i.e.,

$$Q_b = 3.1 B_r h_{\text{weir}}^{3/2} \quad (5)$$

in which B_r is the breach width (ft) and h_{weir} is the instantaneous head (ft) over the weir.

In the finite time interval it takes for the breach to form, the volume of water that flows out of the reservoir is the integral of the instantaneous flow (Q_b) over the time interval from zero to t_f where t_f is the time of failure. This outflow volume may also be expressed as the reservoir surface area (A_s) (assumed constant during the time of failure) multiplied by the integral of the instantaneous drawdown (y_d) over the total change in pool level (y_f) during time of failure. Using the above expression for Q_b (Eq. 5), and equating these two expressions for the outflow volume, the following basic relation is developed:

$$3.1 B_r \int_0^{t_f} h_{\text{weir}}^{3/2} dt = A_s \int_0^{y_f} dy_d \quad (6)$$

The instantaneous head (h_{weir}) over the weir (see Fig. 2) is

$$h_{\text{weir}} = (H - y_b) - y_d \quad (7)$$

where y_b is the instantaneous height of the bottom of the breach.

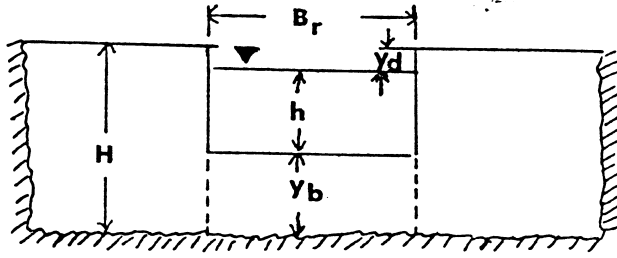


Fig.2 Instantaneous Breach Geometry at
 $0 < t < t_f$

Substituting into Eq. (6) the expression for h_{weir} given in Eq. (7) yields a relation that cannot be integrated analytically. To obtain an analytical solution, it is necessary to neglect the instantaneous height of the breach (y_b) in Eq. (7) and approximate h_{weir} as follows:

$$h_{weir} = H - y_d \quad (8)$$

This assumption of the negligible effect of y_b has been shown to be acceptable by test comparisons of the peak outflow computed by the NWS DAMBRK Model for a wide range of dam and reservoir sizes and failure times. (However, in a rare case where a dam impounding a small storage volume has a large time of failure, the equation developed in this section for calculating maximum breach outflow will predict much higher flow than actually occurs.) Upon substituting the approximate expression for h_{weir} into Eq. (6) and rearranging terms, the following analytically solvable relation is obtained:

$$\int_0^{t_f} dt = \frac{A_s}{3.1 B_r} \int_0^{y_f} \frac{dy_d}{(H - y_d)^{3/2}} \quad (9)$$

Integrating this and evaluating at the limits yields:

$$t_f = \frac{C}{\sqrt{H - y_f}} - \frac{C}{\sqrt{H}} \quad (10)$$

where, for A_s expressed in acres and t_f in hours:

$$C = \frac{23.4 A_s}{B_r} \quad (11)$$

Rearranging Eq. (10) to solve for $(H - y_f)$ yields the following:

$$(H - y_f) = \left(\frac{C}{t_f + \frac{C}{\sqrt{H}}} \right)^2 \equiv h_{weir}^{(max)} \quad (12)$$

As indicated, this equation gives the maximum head over the weir at $t = t_f$. Thus, the maximum breach outflow (Q_{bmax}) occurring at $t = t_f$ may be calculated by substituting this expression for $h_{weir}^{(max)}$ into Eq. (5), i.e.,

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Once the maximum outflow has been calculated, the maximum stage (h_{max}) at the downstream face of the dam must be determined. To do this, the flow (Q_v) that produces the stage (h_v) shown in Fig. 3 must first be evaluated and compared with Q_{bmax} to determine whether the maximum stage is above or below h_v . The flow Q_v is evaluated using the Manning equation, i.e.,

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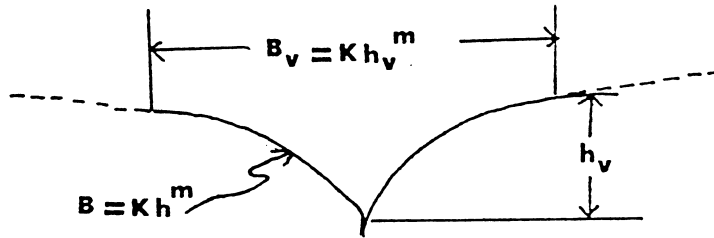


Fig.3 Calculation of Flow (Q_v) For Determining Maximum Stage

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where:

$$a = \frac{1.49}{n} S^{1/2} \frac{K}{(m+1)^{5/3}} \quad (17)$$

The distinguishing characteristic of each curve family is the Froude number developed as the floodwave moves downstream. The distinguishing characteristic of each member of a family is the ratio of the volume in the reservoir to the average flow volume in the downstream channel. Thus it may be seen that to predict the peak flow and travel-time of the floodwave at a downstream point the user must first determine the desired distinguishing characteristic of the curve family and member. This determination is based on the calculation of the Froude number and the volume ratio parameter. To specify the distance and travel time in dimensionless form, the distance and time parameters mentioned earlier must also be computed.

2.5.1 Routing Parameters

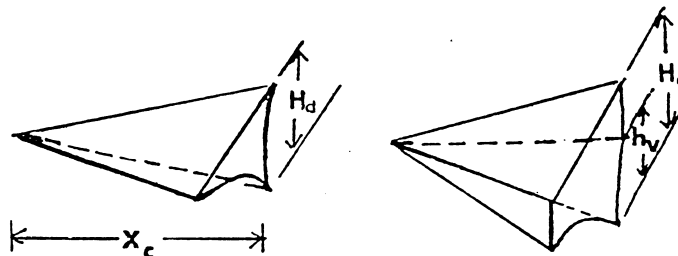
The distance parameter, X_c , may be conceptualized as the length of a pyramid shaped reservoir that is equivalent in volume to the actual reservoir being analyzed (see Fig. 4a and b). For a reservoir in which the initial pool elevation is below h_v , X_c (in ft) is determined using the relation:

$$X_c = \frac{2(m+1) \text{VOL}_R}{K H_d^{m+1}} \quad (30)$$

in which VOL_R is the reservoir volume (ft^3), H_d is the height (ft) of the dam, and K and m are the fitted channel description parameters.

If the initial pool elevation is above h_v (see Fig. 4b), X_c is defined by:

$$X_c = \frac{2 \text{VOL}_R}{K h_v^m \left(\frac{h_v}{m+1} + H_d - h_v \right)} \quad (31)$$



CASE I $H_d \leq h_v$

Fig. 4a

CASE II $H_d > h_v$

Fig. 4b

Distance Parameter (X_c) Calculation

Within the distance (X_c) in the downstream reach, the floodwave attenuates such that the stage at the point X_c is h_x (see Fig. 5),

$$b = m + 5/3 \quad (18)$$

If however, Q_v is found to be less than Q_{bmax} then h_v must be less than h_{max} , and h_{max} is calculated as follows:

$$h_{max} = \rho Q_{bmax}^{3/5} + \gamma h_v \quad (19)$$

where:

$$\rho = \left[\frac{1}{a(m+1)^{5/3} h_v^m} \right]^{3/5} \quad (20)$$

$$\gamma = \frac{m}{m+1} \quad (21)$$

2.4.1 Submergence Correction

The maximum stage produced by the breach outflow must be compared with the head over the weir to find whether it is necessary to include a submergence correction for tailwater effects on the breach outflow. If the computed h_{max} is greater than $(0.67 h_{weir})$ where h_{weir} is the final head over the weir (breach) as expressed in Eq. (12), the maximum breach outflow must be corrected for submergence. This correction factor, k_s , is given by:

$$k_s = 1 - 27.8 \left[\frac{h_{max}}{h_{weir}} - 0.67 \right]^3 \quad (22)$$

The breach outflow is corrected for submergence using the following relation:

$$Q_b^* = k_s^* Q_{bmax} \quad (23)$$

$$k_s^* = \frac{1 + k_s}{2} \quad (24)$$

in which k_s^* is an average submergence correction factor from Eq. (22). This averaging anticipates lesser submergence effects due to the resulting reduced outflow. The new maximum breach outflow (Q_b^*) is then compared with Q_v , and a new outflow stage (h_{max}) is calculated using Eq. 16 or 19. Also, because there is decreased flow through the breach, there is less drawdown. Thus, the head over the weir (h_{weir}) must be recalculated using the relation:

$$h_{weir}^* = h_{weir} + \left(Q_{bmax} - Q_b^* \right) \frac{t_f \text{ (sec.)}}{2A_s \text{ (sq.ft.)}} \quad (25)$$

Now the ratio of the two new values, h_{max}/h_{weir} is used in Eq. (22)

to compute a new submergence correction factor. If k_s is less than 1.0, submergence is still affecting flow and a Newton-Raphson iteration scheme may be employed to adjust the calculated value of Q_b^* . The Newton-Raphson iterative equation is:

$$Q_b^{k+1} = Q_b^k - \frac{f(Q^k)}{f'(Q^k)} \quad (26)$$

where the k superscript represents the iteration counter, and $f'(Q^k)$ represents the derivative of $f(Q^k)$ with respect to Q . In expanded form, $f(Q^k)$ is given by:

$$f(Q^k) = Q_b^k - \left[1 - 27.8 \left(\frac{h_{\max}}{h_{\text{weir}}} - 0.67 \right) \right]^3 \left(3.1 B_r h_{\text{weir}}^{3/2} \right) \quad (27)$$

If Eq. (16) was used to compute h_{\max} (e.g., $Q_b < Q_v$), the derivative of $f(Q^k)$ is:

$$f'(Q^k) = 1 + \left(3.1 B_r h_{\text{weir}}^{3/2} \right) (83.4) \left[\frac{\left(\frac{Q_b^k}{a} \right)^{1/b}}{h_{\text{weir}}} - 0.67 \right]^2 \quad (28)$$

If Eq. (19) was used to compute h_{\max} , then the derivative of $f(Q^k)$ is:

$$f'(Q^k) = 1 + \left(3.1 B_r h_{\text{weir}}^{3/2} \right) (83.4) \left[\frac{\rho (Q_b^k)^{3/5} + \gamma h_v}{h_{\text{weir}}} - 0.67 \right]^2 \quad (29)$$

where ρ and γ are defined by Eqs. (20-21).

Substituting the evaluated $f(Q^k)$ and $f'(Q^k)$ into Eq. (26) gives an improved estimate of the maximum breach outflow. Within one or two iterations a suitable value for the maximum breach outflow is achieved which properly accounts for the effects of submergence.

2.5 Downstream Routing

After the maximum breach outflow and stage have been calculated, it is necessary to route the flow downstream. This routing is achieved by employing dimensionless curves developed using the NWS DAMBRK Model. These dimensionless curves are grouped into families (see Appendix I) and have as their X-coordinate the ratio of the downstream distance of forecast points to a distance parameter discussed in the following section. The Y-coordinate of the curves used in predicting peak downstream flows is the ratio of $(Q_{\text{peak}}/Q_{b\text{max}})$, while the Y-coordinate of the curves for predicting travel times is the ratio of the travel time to a time parameter discussed in the following section.

which is a function of the maximum stage (h_{\max}). The average stage (\bar{h}) in this reach is:

$$\bar{h} = \frac{(h_{\max} + h_x)}{2} = \theta h_{\max} \quad (32)$$

where θ is an empirical weighting factor.

For reaches in which the X_c value is less than 10 miles, the θ value is generally between 0.85 and 0.95, while for larger X_c values the θ value is between 0.5 and 0.85.

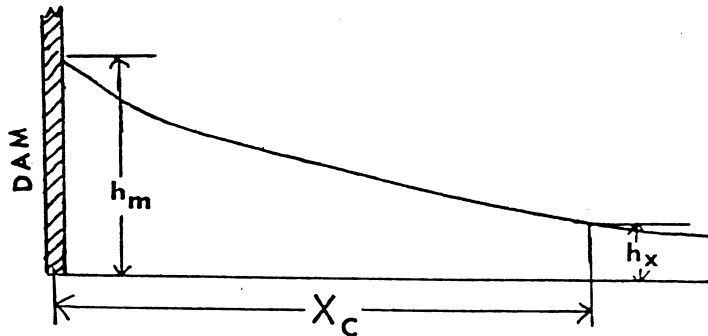


Fig.5 Floodwave Stage Attenuation

The average hydraulic depth (D_c) within the X_c reach is:

$$D_c = \frac{A}{B} = \frac{\left(\frac{K(\theta h_{\max})^{m+1}}{(m+1)} \right)}{K(\theta h_{\max})^m} = \frac{\theta h_{\max}}{m+1} \quad (33)$$

Then, from the Manning equation, the average velocity (in ft/sec) within the X_c reach is given by:

$$V_c = \frac{1.49}{n} S^{1/2} \left(\frac{\theta h_{\max}}{m+1} \right)^{2/3} \quad (34)$$

The time parameter (T_c) is simply a measure of the time it takes the wave to travel through the X_c reach and is given by:

$$T_c = \frac{X_c}{V_c} \quad (35)$$

The average Froude number in the X_c reach is given by:

$$F_c = \frac{V_c}{\sqrt{gD_c}} \quad (36)$$

This data is substituted into Eq. (11) to yield:

$$C = \frac{23.4 (1936)}{150} = 302$$

Entering this value for C into Eq. (13) gives the following value for the maximum breach outflow:

$$Q_{bmax} = 3.1 (150) \left(\frac{302}{1.25 + \frac{302}{\sqrt{261.5}}} \right)^3 = 1,619,025 \text{ cfs}$$

To determine the maximum outflow stage, the value of Q_v must first be calculated. The slope of the downstream channel is 12.5 ft/mi and the estimated Manning's n value is 0.045. These data, as well as the fitted values for K & m, are substituted into Eq. (15) to yield:

$$Q_v = \frac{1.49}{.045} \left(\frac{12.5}{5280} \right)^{1/2} \left(\frac{58.9}{(.66+1)^{5/3}} \right) (25)^{(.66+5/3)} = 148,537 \text{ cfs}$$

Because Q_{bmax} is greater than Q_v , the maximum outflow stage (h_{max}) must be greater than h_v . The value of h_{max} is found using Eq. (19), i.e.,

$$h_{max} = \rho Q_{bmax}^{3/5} + \gamma h_v = (.0111) (1,619,025)^{3/5} + (.398) 25 = 68.7 \text{ ft}$$

$$\gamma = \frac{m}{m+1} = \frac{.66}{1.66} = .398$$

$$\rho = \left[\frac{1}{a(m+1)^{5/3} h_v^m} \right]^{3/5} = \left[\frac{1}{93.44(1.66)^{5/3} (25)^{.66}} \right]^{3/5} = .0111$$

in which:

$$a = \frac{1.49}{n} S^{1/2} \frac{K}{(m+1)^{5/3}} = \frac{1.49}{.045} \left(\frac{12.5}{5280} \right)^{1/2} \frac{135}{(.66+1)^{5/3}} = 93.44$$

Next, the ratio of h_{weir}/h_{max} must be checked to determine whether submergence affects the flow through the weir (breach). Eq. (12) is used to compute h_{weir} , i.e.,

$$h_{weir} = \left(\frac{C}{t_f + \frac{C}{\sqrt{H}}} \right)^2 = \left(\frac{302}{1.25 + \frac{302}{\sqrt{261.5}}} \right)^2 = 229.7 \text{ ft}$$

in which g is the acceleration of gravity (ft/sec^2) and D_c is the average hydraulic depth given by Eq. (33).

The dimensionless volume parameter (V^*) is a ratio of the reservoir storage volume to the average flow volume in the X_c reach. The cross-section area (A_c) associated with the average hydraulic depth (D_c) is:

$$A_c = \frac{K(D_c)^{m+1}}{m+1} \quad (37)$$

Dividing the reservoir storage volume (VOL_R) by the average flow volume ($A_c X_c$) gives the dimensionless volume parameter (V^*), i.e.:

$$V^* = \frac{\text{VOL}_R}{A_c X_c} \quad (38)$$

2.5.2 Routing Curves

Knowing the value of F_c and V^* , the user may consult the specific curve (see Appendix I) defined for these values and check the chosen value of θ used in Eqs. (32-34). The ordinate of the routing curve at ($X/X_c = 1$) is the ratio of Q_p (at X_c) to $Q_{b\max}$. Knowing Q_p , the user may determine the stage (h_x) at X_c using Eqs. (16) or (19). Then θ is checked by using Eq. (32), i.e., $\theta = h/h_{\max}$ where h is computed via Eq. (32). If there is a significant difference in the new value of θ from the initial θ (e.g., $\pm 10\%$), Eqs. (33-38) should be recalculated and the new value of θ rechecked.

Knowing the proper routing curve, the user may then non-dimensionalize the distance downstream to forecast points using the relation:

$$X_i^* = \frac{X_i}{X_c} \quad (39)$$

where X_i is the downstream distance to the i^{th} forecast point, $i = 1, 2, 3, \dots$

The routing curve is consulted to find the value of the ratio of $Q_{\text{peak}}/Q_{p\max}$ and the ratio of the travel time to T_c . The predicted peak flow is then compared to Q_v (Eq. 15) and the peak stage at the i^{th} forecast point is determined using Eq. 16 or 19.

III MODEL TESTING

The SMPDBK Model has been tested with good results on several theoretical dam-break floods to compare its predictive ability with that of the NWS DAMBRK Model. In addition, the model has been tested on the Teton Dam and Buffalo Creek coal-waste dam failures

to determine its ability to reconstruct observed downstream peak stages, discharges and travel times. To further clarify the readers' understanding of how to use the model, the step-by-step procedure using real data from the Teton Dam failure is presented in the following section.

3.1 Teton Dam Failure

The Teton Dam failed on June 5, 1976, killing 11 people and causing \$400 million in damages to the downstream Teton-Snake River Valley. The U.S.G.S. report by Ray, et al. (1977) provided observations on breach development, reservoir characteristics, downstream cross-sections and estimates of Manning's n, indirect peak discharge measurements at three sites, flood-peak elevations and travel times.

From topographic maps, the following table of topwidth vs. h (where h is the elevation above the channel invert) was developed:

TABLE 1--Cross-section Table for Channel Downstream of Teton Dam

Mile 0.0		Mile 5.0		Mile 8.5	
h	B	h	B	h	B
0.	0.	0.	0.	0.	0.
10.	590.	10.	570.	10.	800.
24.	820.	24.	914.	24.	2000.
80.	1130.	50.	1100.	33.	11000.
85.	1200.	55.	1200.	38.	15000.

With this data, a single, distance weighted cross-section may be developed using Eq. 1., i.e., for h = 10,

$$\bar{B} = \frac{\left(\frac{590+570}{2}\right) (5.0-0.0) + \left(\frac{570+800}{2}\right) (8.5-5.0)}{(8.5-0.0)} = 623.25 \text{ ft}$$

For h = 24, the distance weighted topwidth is 1110 ft.

From topo maps it appears that a reasonable value for h_v is 25 ft. Thus the parameters K and m used in describing the channel as a prism following the relation $B = Kh^m$ must be fitted to the weighted cross-section developed above. Substituting these values into Eqs. (2) and (3) and working through the algorithm yields $K = 135$ and $m = 0.66$.

To compute the maximum breach outflow, the following data was obtained:

Reservoir Volume:	230,473 acre-ft
Reservoir Surface Area at Failure (A_s):	1936 acres
Breach Width (B_r):	150 ft
Breach Height (\bar{H}):	261.5 ft
Time of Failure (t_f):	1.25 hr

This data is substituted into Eq. (11) to yield:

$$C = \frac{23.4 (1936)}{150} = 302$$

Entering this value for C into Eq. (13) gives the following value for the maximum breach outflow:

$$Q_{bmax} = 3.1 (150) \left(\frac{302}{1.25 + \frac{302}{\sqrt{261.5}}} \right)^3 = 1,619,025 \text{ cfs}$$

To determine the maximum outflow stage, the value of Q_v must first be calculated. The slope of the downstream channel is 12.5 ft/mi and the estimated Manning's n value is 0.045. These data, as well as the fitted values for K & m, are substituted into Eq. (15) to yield:

$$Q_v = \frac{1.49}{.045} \left(\frac{12.5}{5280} \right)^{1/2} \left(\frac{58.9}{(.66+1)^{5/3}} \right) (25)^{(.66+5/3)} = 148,537 \text{ cfs}$$

Because Q_{bmax} is greater than Q_v , the maximum outflow stage (h_{max}) must be greater than h_v . The value of h_{max} is found using Eq. (19), i.e.,

$$h_{max} = \rho Q_{bmax}^{3/5} + \gamma h_v = (.0111) (1,619,025)^{3/5} + (.398) 25 = 68.7 \text{ ft}$$

$$\gamma = \frac{m}{m+1} = \frac{.66}{1.66} = .398$$

$$\rho = \left[\frac{1}{a(m+1)^{5/3} h_v^m} \right]^{3/5} = \left[\frac{1}{93.44(1.66)^{5/3} (25)^{.66}} \right]^{3/5} = .0111$$

in which:

$$a = \frac{1.49}{n} S^{1/2} \frac{K}{(m+1)^{5/3}} = \frac{1.49}{.045} \left(\frac{12.5}{5280} \right)^{1/2} \frac{135}{(.66+1)^{5/3}} = 93.44$$

Next, the ratio of h_{weir}/h_{max} must be checked to determine whether submergence affects the flow through the weir (breach). Eq. (12) is used to compute h_{weir} , i.e.,

$$h_{weir} = \left(\frac{C}{t_f + \frac{C}{\sqrt{H}}} \right)^2 = \left(\frac{302}{1.25 + \frac{302}{\sqrt{261.5}}} \right)^2 = 229.7 \text{ ft}$$

Because

$$(h_{\max}/h_{\text{weir}}) = (68.7/229.7) = 0.3 < 0.67$$

submergence of the breach does not occur.

Now that the maximum breach outflow discharge and stage have been determined, the routing parameters must be calculated. As the height of the dam (H_d) is greater than h_v , Eq. (31) is used to calculate X_c , i.e.,

$$X_c = \frac{2 \text{ VOL}_R}{K h_v^m \left(\frac{h_v}{m+1} + H_d - h_v \right)} = \frac{2 (230473) (43560 \text{ ft}^2/\text{acre})}{135 (25)^{.66} \left(\frac{25}{1.66} + 261.5 - 25 \right)}$$

$$= 70,652 \text{ ft} = 13.4 \text{ miles}$$

In an X_c reach that is this long, the attenuation of the flood peak will probably be substantial. For this reason a θ value of 0.8 is chosen. This θ is substituted into Eqs. (33) and (34) to determine the average hydraulic depth and velocity in the X_c reach.

$$D_c = \frac{\theta h_{\max}}{m+1} = \frac{0.8 (68.7)}{(.66+1)} = 33.11 \text{ ft}$$

$$V_c = \frac{1.49}{n} S^{1/2} D_c^{2/3} = \frac{1.49}{.045} \left(\frac{12.5}{5280} \right)^{1/2} (33.11)^{2/3} = 16.61 \text{ ft/sec}$$

The time parameter (T_c) may now be calculated using Eq. (35), i.e.,

$$T_c = \frac{X_c}{V_c} = \frac{70652}{16.61} = 4253 \text{ sec} = 1.18 \text{ hrs}$$

The Froude number is given by Eq. (36), i.e.,

$$F_c = \frac{V_c}{\sqrt{g D_c}} = \frac{16.61}{\sqrt{32.2(33.11)}} = 0.5$$

The cross-sectional area associated with the average hydraulic depth (D_c) is given by Eq. (37) as follows:

$$A_c = \frac{K (D_c)^{m+1}}{m+1} = \frac{135 (33.11)^{1.66}}{1.66} = 27,124.26 \text{ ft}^2$$

Multiplying A_c by the distance parameter (X_c) and dividing the product into the reservoir volume (VOL_R) gives the dimensionless volume parameter as in Eq. (38), i.e.,

parameters corresponding to each sub-reach, the user may forecast downstream flooding for as far as desired regardless of what changes occur in the valley geometry.

3.2 Buffalo Creek Flood

The most catastrophic flood in West Virginia's history occurred on February 26, 1972 when the collapse of the Buffalo Creek coal-waste dam released over 400 acre-ft of water and coal sludge in 15 minutes, killing 118 people and destroying \$50 million worth of property. Two separate reports by the U.S. Geological Survey (Davies, et al., 1972) and (McQuivey and Keefer, 1975) provided the input data required by the SMPDBK Model and the observed data necessary to examine the model's accuracy.

Fitting a prismatic channel to the cross-section data provided in the reports gives the channel description parameters of $K = 60.$, $m = 0.58$, and $h_v = 10$ ft. Substituting the breach data of $t_f = 0.083$ hrs, $B_r = 274$ ft (trapezoidal), $H = 40$ ft, and $A_s = 13.1$ acres into Eq. (13), provides a value for Q_{bmax} at the dam of 62,700 cfs which produces a stage (h_{max}) of 17 ft. Although the channel slope immediately downstream (3.6 miles) of the dam is supercritical, the average slope (53 ft/mi) and roughness ($n = .052$) of the full 12.1 mile reach is used in this investigation to compute the following routing parameters: $X_c = 0.69$ mi, $\theta = 0.95$, $T_c = 0.07$ hrs, $F_c = 0.78$, and $V^* = 2.26$. The McQuivey report presents the observed flows and travel times at 6.8 and 12.1 miles downstream of the dam. Thus, the chart for $F_c = 0.75$ is consulted to find the ratios of Q_p/Q_{bmax} at $X/X_c = 6.8/0.69 = 9.8$ and $X/X_c = 17.53$. (Unfortunately, because of limited space in this paper, the entire curves can not be presented in Appendix I. However, an upcoming NWS publication will present the entire routing curve families for a wide range of F_c values.) From the chart, the Q_p/Q_{bmax} ratio at $X/X_c = 9.8$ is found to be 0.27, indicating a flow of 16,929 cfs which produces a stage of 11.19 ft. The T_p/T_c curve shows the time of travel to mile 6.8 to be 0.8 hrs. These results compare reasonably well with the indirect USGS measurements of $Q = 13,000$ cfs (30% error), stage = 10 ft (1.19 ft error) and travel time = .91 hours (13% error). At mile 12.1 ($X/X_c = 17.5$), the Q_p/Q_{bmax} ratio is 0.19 which, when multiplied by Q_{bmax} , gives a flow of 11,913 cfs. This flow produces a stage of 11.96 ft in the narrow cross-section at mile 12.1 and has a travel time of 1.45 hrs. Here the model results are somewhat less accurate as the measured flow was 8,800 cfs (35% error), the observed stage was 10.5 ft (1.46 ft error), and the travel time was 2 hours (25% error).

It should be stressed that although the accuracy of SMPDBK in this test comparison appears less than desirable, the "observed" measurements may be somewhat suspect as they are based on eyewitness accounts and estimates of channel geometry and roughness used in conjunction with high-water marks. Another factor that reduces the accuracy of the model is the substantial volume of sludge that was released in the dam failure. As the flow velocities decreased, this

$$V^* = \frac{VOL_R}{A_c X_c} = \frac{(230,473) (43,560)}{(27,124) (70,652)} = 5.24$$

All of the required parameters have now been calculated, and the initial estimate of θ may be checked. To recap, the following is a list of the parameters that have been calculated:

$$X_c = 13.4, T_c = 1.18, F_c = 0.5, V^* = 5.24$$

To check the value of θ , the family of curves for $F_c = 0.5$ in Appendix I is consulted. Moving vertically from the point where $X/X_c = 1.0$ to a point interpolated between the curves for $V^* = 4.0$ and $V^* = 7.0$ corresponding with $V^* = 5.24$, the ordinate of this point is found to equal 0.5, indicating that at X_c , the peak flow has attenuated to (.5) (Q_{bmax}).

$$Q_p (X = X_c) = .5 (1,619,025) = 809,512. \text{ cfs}$$

This flow is greater than Q_v ($Q_v = 148,537$) so Eq. (19) is used to determine the maximum stage at X_c , i.e.,

$$h_x = \rho Q_p^{3/5} + \gamma h_v = (.0111) (809,512)^{3/5} + (.398) 25 = 48.53$$

To check the value of θ , Eq. (33) is rearranged such that:

$$\theta = \frac{\frac{h_{max} + h_x}{2 h_{max}}}{2 (68.7)} = \frac{68.7 + 48.53}{2 (68.7)} = 0.84$$

This indicates that the original estimate of $\theta = 0.8$ is acceptable.

Now the peak flow, stage, and travel time at the forecast point ($X = 8.5$ miles) may be determined. Interpolating between the ($V^* = 4.0$) and ($V^* = 7.0$) curves at $X/X_c = 8.5/13.4 = 0.63$, the ratio of Q_p/Q_{bmax} is found to be 0.59. Thus,

$$Q_p = 0.59 (1,619,025) = 957,249 \text{ cfs}$$

To most accurately predict the stage at $X = 8.5$, the distance weighted cross-section is put aside in favor of the true cross-section and Manning's n at $X = 8.5$ miles. The following B vs. h table corresponds to the cross-section at $X = 8.5$:

TABLE 2--Cross-Section Properties for Mile 8.5

h	B
0.	0.
10.	800.
24.	2000.
33.	11000.
38.	15000.

Manning's n at mile 8.5 is estimated to be .037. As can be seen from the table, the valley at mile 8.5 is substantially wider than in the reach just below the dam. Indeed, no h_v can be discerned from the data. Thus, all the data points will be used in fitting the K and m parameters to describe this cross-section. Substituting this data into Eqs. (2-4) yields $K = 4.26$ and $m = 2.17$.

If there is an h_v value, it is very large, and it is safe to assume that Q_p is less than Q_v . Because $Q_p < Q_v$, Eq. 16 is used to calculate the peak stage at $X = 8.5$ miles, i.e.,

$$a = \frac{1.49}{n} S^{1/2} \frac{K}{(m+1)^{5/3}} = 1.22$$

$$b = m + 5/3 = 3.837$$

$$h_p = \left(\frac{Q_p}{a} \right)^{1/b} = 34.4 \text{ ft}$$

The time to peak at mile 8.5 is found by moving vertically from $X/X_c = .63$ and interpolating ($V^* = 5.24$) between the ($V^* = 4.0$) and ($V^* = 7.0$) curves of the (T_p/T_c) vs. (X/X_c) chart for $F_c = 0.5$. From this chart, the value of T_p/T_c is found to be 0.64. Multiplying this value by T_c and adding this product to the time of failure (t_f) yields

$$\text{Time to peak} = 0.64 (T_c) + t_f = 0.64 (1.18) + 1.25 = 2.0 \text{ hrs}$$

To examine the prediction accuracy of the SMPDBK Model, the computed forecast must now be compared with the observed data provided in the Geological Survey Report (Ray, et al., 1977). This report indicates that at mile 8.5 the observed discharge was 1,060,000 cfs. This compares well with the SMPDBK Model's predicted flow of 957,249 cfs for an error of less than 10%. The model produced an estimated flow depth of 34.4 ft at mile 8.5, which, when added to the bottom elevation of 4920., gives a water surface elevation of 4954.4. This also compares well with the observed stage of 4953 at mile 8.5. The time of peak discharge at mile 8.5 "probably occurred between 1300 and 1400 hours" (1 to 2 hours after failure began) according to the report. Test runs of the NWS DAMBRK Model indicate that this time is probably closer to two hours which is exactly the time predicted by the SMPDBK Model.

As can be noted from the above comparison, the SMPDBK model produces very good results in forecasting the downstream flooding produced by the Teton failure. Unfortunately, because the river valley widened so greatly after mile 8.5, further downstream routing is not possible with the model's present capabilities. However, further development is currently under way to allow the user to divide the downstream reach into smaller reaches, each having its own specific characteristics. Based on the Froude and volume

parameters corresponding to each sub-reach, the user may forecast downstream flooding for as far as desired regardless of what changes occur in the valley geometry.

3.2 Buffalo Creek Flood

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It should be stressed that although the accuracy of SMPDBK in this test comparison appears less than desirable, the "observed" measurements may be somewhat suspect as they are based on eyewitness accounts and estimates of channel geometry and roughness used in conjunction with high-water marks. Another factor that reduces the accuracy of the model is the substantial volume of sludge that was released in the dam failure. As the flow velocities decreased, this

sludge settled out, thereby reducing the effective volume flowing down the channel. In a previous study of the Buffalo Creek flood, calibration of the DAMBRK Model indicated that this volume loss reduced the peak flow by 7% at mile 6.8 and by 13% at mile 12.1. Applying these volume loss corrections reduces the flow at mile 6.8 to 15,700 cfs (20% error), which produces a stage of 10.9 ft (0.9 ft error). At mile 12.1 the flow is reduced to 10,364 cfs (17% error) and the stage is lowered to 11.66 ft (1.16 ft error). There was also a considerable amount of off-channel storage in the downstream reach that is not fully accounted for in the simplified model. This storage further reduced the peak flows and increased the travel times.

IV CONCLUDING REMARKS

There is a clear need for real-time forecasts of dam-break floodwave peak stages and travel times. A flash flood hydrologist employing the NWS SMPDBK Model can provide such forecasts quickly and with reasonable accuracy. For example, in each of the test cases presented in this paper, approximating the channel as a prism, calculating the maximum breach outflow and stage at the dam, defining the routing parameters, and evaluating the peak stage and travel time to the forecast points, required less than 20 minutes of time with the aid of a non-programable hand-held calculator while the average error in forecasted peak flow and travel time was 10-20% with stage errors of approximately 1 ft. Furthermore, comparisons of SMPDBK Model results with DAMBRK Model results from test runs of theoretical dambreaks show the simplified model generally produces errors of less than 10%. The authors had the advantages, however, of prior experience with the model and possession of all required input data, the collection of which consumes precious warning response time in a dam-break emergency.

To help reduce the time required for data collection, a table of possible default values for some of the input data is presented in Table 3. These default values may be used by dam-break flood forecasters when time is short and reliable data is unavailable. Additionally, to help further reduce the time needed to employ the model, work is currently under way to code the entire procedure on both a programmable calculator and an NWS AFOS system S230 mini-computer. When this coding is complete, the program will be recorded on magnetic cards and floppy discs that will be disseminated to model users.

TABLE 3--Default Input Data Values

<u>Value</u>	<u>Units</u>	<u>Default</u>	<u>Description</u>
B_r	ft	2 x breach depth	Breach width
t_f	hr	0.1--2.0	Time of failure
h_v	ft	H/2	Elevation of valley wall base or thick vegetation
n	--	0.06	Downstream channel roughness coefficient
S_o	--	Dam Height/ Reservoir length	Bottom slope of downstream channel

The SMPDBK Model is not only useful in a dam-break emergency, it is also suitable for pre-computation of flood peak elevations and travel times prior to a dam failure. Pre-computation of dam failures allows those responsible for community preparedness to delineate danger areas downstream should the dam fail. Ideally, the sophisticated NWS DAMBRK model would be used in a long-term disaster preparedness study with sufficient computer resources to obtain the most reliable estimate of probable flood elevations and travel times. However, for short term studies with limited resources and relaxed accuracy requirements, the SMPDBK Model will be most helpful in defining peak stages, discharges, and travel times.

Although the model in its present form is a useful tool for dam-break flood forecasting, it does have limitations which reduce its accuracy. As was seen in the presentation of the Teton flood study, the model was not capable of routing the flood past mile 8.5 because the valley geometry changed so drastically downstream of that point. Also, the accuracy of the results in the Buffalo Creek study would probably have improved had the authors been able to subdivide the downstream channel into the 3.6 mile supercritical reach and the 8.5 mile subcritical reach. As was indicated earlier, however, research is under way to develop a method that will allow such a subdivision of the downstream channel. Work is also being performed to allow the model to compensate for special downstream valley characteristics, such as off-channel storage and/or bridge-road embankments, that are neglected in the prismatic channel approximation. With these improvements, the NWS SMPDBK model has the potential for producing very reliable results while consuming little of the users' time and resources.

REFERENCES

- Davies, W.E., J.F. Bailey, and D.B. Kelly, 1972: "West Virginia's Buffalo Creek Flood: A Study of the Hydrology and Engineering Geology." Geological Survey, Circular 667, U.S. Geological Survey, Washington, D.C., 1972, 32 pp.
- Fread, D.L., 1977: "The Development and Testing of a Dam-Break Flood Forecasting Model." Proceedings, Dam-Break Flood Modelling Workshop, U.S. Water Resources Council, Washington, D.C., 1977, pp. 164-197.
- Fread, D.L., 1980: "DAMBRK: The NWS Dam-Break Flood Forecasting Model." Office of Hydrology, National Weather Service (NWS), Silver Spring, Md., 1980, 37 pp.
- McQuivey, R.S. and T.N. Keefer, 1975: "Application of a Simple Dambreak Routing Model." Proceedings, 16th Congress IAHR, Vol. II, Sao Paulo, Brazil, 1975, pp. 315-324.
- Ray, H.A., L.C. Kjelstrom, E.G. Crosthwaite, and W.H. Low, 1976: "The Flood in Southeastern Idaho from the Teton Dam Failure of June 5, 1976." Unpublished open file report, U.S. Geological Survey, Boise, Idaho.
- Saakas, J.G., and T. Strelkoff, 1973: "Dam-Break Flood in a Prismatic Dry Channel." Journal of the Hydraulics Division, ASCE, Vol. 99, No. HY12, Proc. Paper 10233, Dec. 1973, pp. 2195-2216.

- SCS, 1979: "Simplified Dam-Breach Flood Routing Procedure."
Technical Release No. 66, Engineering Division, Soil Conservation
Service, 1979, 23 pp.
- Snyder, F.F., 1977: "Floods from Breaching of Dams." Proceedings,
Dam-Break Flood Modelling Workshop, U.S. Water Resources Council,
Washington, D.C., 1977, pp. 75-85.
- U.S. Army Corps of Engineers, 1975: National Program of Inspection
of Dams, Vol. I-V, Dept. of the Army, Office of Chief of
Engineers, Washington, D.C.

